Section 7: Commentary
SEISMIC DESIGN REQUIREMENTS (SDR) 3

C7.2 DESIGN FORCES

C7.2.1 Ductile Substructures (R>1) — Flexural Capacity

The key element in the design procedure is the flexural capacity of the columns. Philosophically the lower the flexural capacity of the column the more economic the seismic design provisions because the overstrength flexural capacity of a column drives the cost and capacity of the foundations and the connection to the superstructure. For SDAP B the capacity of the column designed for non-seismic loads is considered to be acceptable for the lower seismic hazard levels.

For SDAP C the design procedure provides a trade-off between acceptable design displacements and minimum flexural capacities of columns. For SDAP D and E the flexural capacity of a column must meet the maximum of the moments from either the 50% PE in 25 years event or the 3% PE in 75 year event divided by the appropriate R-Factor. For SDAP C, D, and E there are additional strength limitations based on P-Δ considerations.

C7.2.2 Capacity Protected Elements or Actions

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top and/or bottom) where they can be readily inspected and/or repaired. To achieve this objective all members connected to the columns, the shear capacity of the column and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements is when all substructure elements are designed elastically (Article 4.10), seismic isolation design (Article 7.10) and in the transverse direction of columns when a ductile diaphragm is used (Article 7.7.8.2)

C7.2.3 Elastically Designed Elements

If all the supporting substructures elements (columns, piers, pile bents) are designed elastically, there will be no redistribution of lateral loads due to plastic hinges developing in one or more columns. As a consequence the elastic analysis results are appropriate for design. The recommended provisions attempt to prevent any brittle modes of failure from occurring.

If only one or a selected number of supporting substructure elements are designed elastically, there will be a significant redistribution of lateral loads when one or more of the columns develop plastic hinges. Generally, the elastically designed elements will attract more lateral load. Hence the need to either use capacity design principles for all elements connected to the elastically designed column. If this is not practical, the complete bridge needs to be reanalyzed using the secant stiffness of any columns in which plastic hinges will form in order to capture the redistribution of lateral loads that will occur.

C.7.2.4 Abutments and Connections

In general the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases where the full plastic mechanism might not develop during the 3% in 75-year earthquake, the elastic forces of this event are permitted. However, it is still good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents the connections should be stronger than the expected forces, and these forces may be quite large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths, rather than the piles' structural strengths.

C7.2.5 Single Span Bridges

Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single span bridges need not be analyzed for seismic loads regardless of the SDR and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures.

C7.3 DESIGN DISPLACEMENTS

See Article C8.3 for the commentary to this section.

C7.4 FOUNDATION DESIGN REQUIREMENTS

C7.4.1 Foundation Investigation

Refer to C8.4.1 for the commentary to this section.

C7.4.2 Spread Footings

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force at the abutments and central piers. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and an overturning moment being imposed on the footing. The footing responds to this load by combined horizontal sliding and rotation. The amount of sliding and rotation depends on the magnitude of imposed load, the size of the footing, and the characteristics of the soil.

For seismic design of spread footings, the response of the footing to shear forces and moment is normally treated independently; i.e., the problem is de-coupled. The overturning component of the column load results in an increase in pressures on the soil. Since the response to moment occurs as a rotation, pressure is highest at the most distant point of the footing, referred to as the toe. This pressure can temporarily exceed the ultimate bearing capacity of the soil. As the overturning moment continues to increase, soil yields at the toe and the heel of the footing can separate from the soil, which is referred to as liftoff of the footing. This liftoff is temporary. As the inertial forces from the earthquake change direction, pressures at the opposite toe increase and, if moments are large enough, liftoff occurs at the opposite side. Bearing failure occurs when the force induced by the moment exceeds the total reactive force that the soil can develop within the area of footing contact. Soil is inherently ductile, and therefore, yielding at the toe and liftoff at the heel of the footing are acceptable phenomena, as long as (1) global stability is preserved and (2) settlements induced by the cyclic loading are small.

The shear component of column load is resisted by two mechanisms: (1) the interface friction between the soil and the footing along the side and at the base of the footing, and (2) the passive resistance at the face of the footing. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it normally provides the primary resistance to horizontal loading.

Inertial response of a bridge deck results in a horizontal shear force and a moment at the connection of the column to the footing. The footing should not undergo permanent rotation, sliding, or appreciable settlement under these loads. Any permanent displacement that occurs should be constrained by the limits required to preserve the service level of the bridge as suggested in Table C3.2-1.

C7.4.2.1 Moment and Shear Capacity

The shear component of loading should not be included during the overturning check; i.e., a de-coupled approach should be used in treating the two loads. Experience has shown that use of inclination factors to represent the combined horizontal load and moment in simplified bearing capacity equations can result in unreasonably sized footings for seismic loading.

Unfactored resistance is used for the moment capacity check for two reasons: (1) the potential for the design seismic load is very small, and (2) the peak load will occur for only a short duration. The distribution and magnitude of bearing stress, as well as liftoff of the footing, are limited to control settlement of the footing from the cycles of load.

Non-triangular stress distributions or greater than 50 percent liftoff are allowed if studies can show that soil settlement from cyclic shakedown does not exceed amounts that result in damage to the bridge or unacceptable movement of the roadway surface. By limiting stress distribution and the liftoff to the specified criteria, the amount of shakedown will normally be small under normal seismic loading conditions.

No special check is required for the shear component of column loads for SDR 3 because the maximum horizontal load induced by the seismic event will normally be less than the friction mobilized at the base of the footing for this seismic category.

C7.4.2.2 Liquefaction Check

Liquefaction below a spread footing foundation can result in three conditions that lead to damage or failure of a bridge:

- loss in bearing support which causes large vertical movement,
- horizontal forces on the footing from lateral flow or lateral spreading of the soil, and
- settlements of the soil as porewater pressures in the liquefied layers dissipate.

Most liquefaction-related damage during past earthquakes has been related to lateral flow or spreading of the soil. In the case of lateral flow and spreading, ground movements could be a meter or more. If the spread footing foundation is located above the water table, as often occurs, it will be very difficult to prevent the footing from being displaced with the moving ground. This could result in severe column distortion and eventual loss of supporting capacity.

In some underwater locations, it is possible that the flowing ground could move past the footing without causing excessive loading; however, these cases will be limited. For these situations special studies are required to evaluate the magnitude of forces that will be imposed on the foundation and to confirm that these forces will not result in large lateral movement of the footing.

Additional discussion of the consequences of liquefaction is provided in Appendix D to these Specifications. A flow chart showing the methodology for addressing the moving soil case is given in Figure D.4.2-1.

C7.4.3 Driven Piles

C7.4.3.1 General

To meet uplift loading requirements during a seismic event or during ship impact, the depth of penetration may have to be greater than minimum requirements for compressive loading to mobilize sufficient uplift resistance. This uplift requirement can impose difficult installation conditions at locations where very hard bearing layers occur close to the ground surface. In these locations ground anchors, insert piles, and H-pile stingers can be used to provide extra uplift resistance in these situations.

If batter piles are used in SDR 3 and above, consideration must be given to (1) downdrag forces caused by dissipation of porewater pressures following liquefaction, (2) the potential for lateral displacement of the soil from liquefaction-induced flow or lateral spreading, (3) the ductility at the connection of the pile to the pile cap, and (4) the buckling of the pile under combined horizontal and vertical loading. These studies will have to be more detailed than those described elsewhere within Article 8.4. As such, use of batter piles should be handled on a case-by-case basis. Close interaction between the geotechnical engineer and the structural engineer will be essential when modeling the response of the batter pile for seismic loading.

Seismic design loads will have a very low probability of occurrence. This low probability normally justifies not using the highest groundwater level during seismic design.

C7.4.3.2 Design Requirements

Shear forces and overturning moments developing within this design category will normally be small. Except in special circumstances, the load and resistance factors associated with Strength Limit State will control the number and size of the pile foundation system. A capacity check under overturning moment is, however, required to confirm that the specific features of the bridge design and soil conditions do not result in instability or excessive uplift of the foundation system. Checks should also be made to confirm that unacceptable displacements from flow slides or loss of bearing support from liquefaction do not occur.

The flexibility of pile bents is included because it is relatively easy to include and it is generally more significant than that of spread and piled foundations. For pile bents the estimated depth of fixity can be determined in one of the following ways: (1) using the simplified relationships shown in Figure C7.4.3.2-1 (Figure 10.7.4-1 of LRFD Provisions) and Figure C7.4.3.2-2 (Figure 10.7.4-2 of LRFD Provisions), (2) using relationships given in FHWA (1997) and DM7 (1982), or (3) conducting lateral pile analyses using a beam-column approach.

C7.4.3.3 Moment and Shear Design

Unfactored resistance and uplift are permitted for the foundation design for two reasons: (1) the design seismic load is likely to be small, and (2) the peak load will occur for only a short duration. By allowing uplift in only the most distant row of piles, the remaining piles will be in compression. Normally piles designed for the Strength Limit State will have a capacity reserve of 2.0 or more, resulting in adequate capacity for vertical loads. The moment capacity check determines whether adequate capacity exists in rotation. If rotational capacities are not satisfied, longer piles or additional piles may be required to meet seismic requirements.

C7.4.3.4 Liquefaction Check

The design of a pile foundation for a liquefied soil condition involves careful consideration on the part of the Designer. Two general cases occur: liquefaction with and without lateral flow and spreading.

<u>Liquefaction without Lateral Flow or Spreading</u>

Pile foundations should be designed to extend below the maximum depth of liquefaction by at least 3 pile diameters or to a depth that axial and lateral capacity are not affected by liquefaction of the overlying layer. Porewater pressures in a liquefied zone can result in increases in porewater within layers below the liquefied zone. Porewater pressures increases can also occur in a zone where the factor of safety for liquefaction is greater than 1.0, as discussed in Appendix D. These increases in porewater pressures will temporarily reduce the strength of the material from its pre-earthquake (static) strength. The potential for this decrease should be evaluated, and the capacity of the foundation evaluated for the lower strength. Alternatively, the toe of the pile should be founded at a depth where the effects of porewater pressure changes are small. Normally, the static design of the pile will include a resistance factor of 0.6 or less. This reserve capacity allows an increase in porewater pressures by 20 percent without significant downward movement of the pile.

As porewater pressures dissipate following liquefaction, drag loads will develop on the side of the pile. The drag loads occur between the pile cap and the bottom of the liquefied layer. The side friction used to compute drag loads will increase with dissipation in porewater pressure from the residual strength of the liquefied sand to a value approaching the static strength of the sand. The maximum drag occurs when the porewater pressures are close to being dissipated. Simultaneously relative movement between the pile and the soil decrease as the porewater pressure decreases, resulting in the drag load evaluation being a relatively complex soil-pile interaction problem. For simplicity, it can be conservatively assumed that the drag load used in the settlement estimate is determined by the pre-liquefied side resistance along the side of the pile between the bottom of the pile cap and the bottom of the liquefied zone.

Liquefaction with Lateral Flow or Spreading

Lateral flow and spreading have been common occurrences during liquefaction at bridge sites involving an approach fill or at a river or stream crossing. The amount of movement can range from a few millimeters to over a meter. This amount of movement is generally sufficient to develop full passive pressures on pile or pile cap surfaces exposed to the moving soil. If the pile-pile cap system is not strong enough to resist these movements, the pile cap system will displace horizontally under the imposed load.

Procedures for estimating either the forces and displacements of the pile from the moving ground are discussed in Appendix D. If these forces or displacements are large, some type of ground remediation might be used to reduce these displacements. These ground remediation methods can include vibro densification, stone columns, pressure grouting, or in-place soil mixing. Costs of these improvements can range from $10/m^3 to in excess of $40/m^3 (in 2000 dollars). Depending on the specific conditions and design requirements for a site, the use of ground improvement could increase construction costs by 10 percent or more. In view of these costs, the Owner needs to be made aware of the potential risks and the costs of remediation methods as soon as these conditions are identified.

Appendix D provides a more detailed discussion of the process to follow when designing for lateral flow or spreading ground.

C7.4.4 Drilled Shafts

Lam et al. (1998) provide a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts, as in the case of driven piles. In this configuration the relative importance of axial and lateral response change. Without the pile cap, lateral-load displacement of the shaft becomes more critical than the axial-load displacement relationships discussed for driven piles.

Many drilled shaft foundation systems consist of a single shaft supporting a column. Compressive and uplift loads on these shafts during seismic loading will normally be within limits of load factors used for gravity loading. However, checks should be performed to confirm that any changes in axial load don't exceed ultimate capacities in uplift or compression. In contrast to driven piles in a group, no reserve capacity exists for a single shaft; i.e., if ultimate capacity is exceeded, large deformations can occur.

Special design studies can be performed to demonstrate that deformations are within acceptable limits if axial loads approach or exceed the ultimate uplift or compressive capacities if the drilled shaft is part of a group. These studies can be conducted using computer programs, such as APILE Plus (Reese, et al., 1997). Such studies generally will require rigorous soil-structure interaction modeling.

Various studies (Lam et al., 1998) have found that conventional p-y stiffnesses derived for driven piles are too soft for drilled shafts. This softer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 600 mm is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the p-y curves. This adjustment is thought to be somewhat dependent on the construction method.

Base shear can also provide significant resistance to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the based of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases the base conditions result in low interface strengths. For this reason the amount of base shear to incorporate in a lateral analyses will vary from case-to-case.

C7.5 ABUTMENT DESIGN REQUIREMENTS

C7.5.1 General

One of the most frequent observations of damage during past earthquakes has been damage to the abutment wall. This damage has been due to two primary causes: (1) the approach fill has moved outward, carrying the abutment with it, and (2) large reactive forces have been imposed on the abutment as the bridge deck has forced it into the approach fill. This latter cause of damage has often resulted from a design philosophy that assumed that the abutment wall had to survive only active seismic earth pressures, and that gaps between the bridge deck and abutment wall would not close. In many cases the gap was not sufficient to remain open, and very large loads were imposed by the deck. The passive reaction from the soil was as much as 30 times the forces used for active pressure design, resulting in overloading to and damage of the wall.

These seismic provisions have been prepared to specifically acknowledge the potential for this higher load to the abutment wall. If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

C7.5.2 Longitudinal Direction

Refer to Article C8.5.2 for the commentary to this subsection.

C7.5.3 Transverse Direction

Refer to Article C8.5.3 for the commentary to this subsection.

C7.6 LIQUEFACTION DESIGN REQUIREMENTS

C7.6.1 General

Liquefaction has been perhaps the single most significant cause of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss in lateral and vertical bearing support of foundations for central piers of a bridge have also occurred.

The potential for liquefaction requires careful attention to the determination of the potential for and consequences of liquefaction. If the mean magnitude of the 3% PE in 75 year event is less than 6.0, then the discussion above with regard to duration is applicable in these SDR's. For the magnitude interval of 6.0 to 6.4, a liquefaction analysis is not required when the combination of ground shaking is below and blow count are above values that would cause liquefaction. This transition interval is based on an assessment of available data from past earthquakes and engineering judgment.

The mean magnitudes shown in Figures 8.6.1-1 to 8.6.1-4 are based on deaggregation information, which can be found in the USGS website (http://geohazards.cr.usgs.gov/eq/). A site-specific determination of the mean magnitude can be obtained from this website using the latitude and longitude of the project site.

If liquefaction occurs in the 50% PE in 75 year event, then the performance criteria for piles will need to be operational for the life safety performance level as per Article 7.8.6.3.

C7.6.2 Evaluation of Liquefaction Potential

Refer to Article C8.6.2 for the commentary to this subsection.

C7.6.3 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

Refer to Article C8.6.3 for the commentary to this subsection.

C7.6.4 Design Requirements if Liquefaction and Ground Movement Occurs

If liquefaction and no lateral flow occur for SDR 3 bridges, then the only additional design requirements are those reinforcement requirements specified for the piles and spread foundation. Additional analyses are not required, although for major or important bridges the additional analyses specified in Article 4.6 may be considered to assess the impact on the substructures above the foundation.

If liquefaction and lateral flow are predicted to occur for SDR 3, a detailed evaluation of the effects of lateral flow on the foundation should be performed. Lateral flow is one of the more difficult issues to address because of the uncertainty in the movements that may occur. The design steps to address lateral flow are given in Appendix D. Note that a liberal plastic rotation of the piles is permitted. This plastic rotation does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur. Design options range from an acceptance of the movements with significant damage to the piles and columns if the movements are large to designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective. Pile group effects are not significant for liquefied soil.

C7.6.5 Detailed Foundation Design Requirements

Refer to the appropriate subsections of Article C7.4 for the commentary.

C7.6.6 Other Collateral Hazards

The assessment of these collateral hazards will normally be limited to bridges located in SDR 3, 4, 5, and 6 as the potential for any of these hazards in SDR 1 and 2 will generally be small.

C7.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

Refer to Article C8.7 for the commentary to all of the subsections of this article.

C7.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

Refer to Article C8.8 for the commentary to all of the subsections of this article.

C7.9 BEARING DESIGN REQUIREMENTS

One of the significant issues that arose during the development of these provisions was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe earthquake, and others that preceded it and have occurred since, clearly showed poor performance of some very recent bearing types and the disastrous consequences that a bearing failure can have on the overall performance of a bridge. A consensus was developed that some testing of bearings would be desirable provided a designer had the option of providing restraints or permitting the bearing to fail if an adequate surface for movement is provided. A classic example occurred in Kobe where a bearing failed and it destroyed the steel diaphragm and steel girder because the girder became jammed on the failed bearing and could not move.

There has been a number of studies performed when girders slide either on specially designed bearings or concrete surfaces. A good summary of the range of the results that can be anticipated from these types of analyses can be found in Dicleli, M., Bruneau, M. (1995).

C7.9.1 Prototype and Quality Control Tests

The types of tests that are required are similar but significantly less extensive than those required for seismically isolated bridges. Each manufacturer is required to conduct a prototype qualification test to qualify a particular bearing type and size for it’s design forces or displacements. This series of tests only needs to be performed once to qualify the bearing type and size, whereas on an isolated project, prototype tests are required on every project. The quality control tests required on 1 out of every 10 bearings is the same as that required for every isolator on seismic isolation bridge projects. The cost of the much more extensive prototype and quality control testing of isolation bearings is approximately 10 to 15% of the total bearing cost, which is of the order of 2% of the total bridge cost. The testing proposed herein is much less stringent than that required for isolation bearings and is expected to be less than 0.1% of the total bridge cost. However, the benefits of testing are considered to be significant since owners would have a much higher degree of confidence that each new bearing will perform as designed during an earthquake. The testing capability exists to do these tests on full size bearings. Caltrans has invested in a full size test machine located at the University of California, San Diego, and similar capabilities exist at other universities, government laboratories, and commercial facilities.

C7.10 SEISMIC ISOLATION DESIGN REQUIREMENT

The commentary on this subject is given in C15 which will become a new section in the AASHTO LRFD provisions.